

Probabilistic assessment of the durability performance of concrete structures

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ABSTRACT

In recent years, much research work has been carried out in order to obtain a more controlled durability and long-term performance of concrete structures in chloride containing environments. In particular, the development of new procedures for probability-based durability design has proved to be very valuable. In order to provide more data and experience with probability-based durability design, two relatively new concrete structures in Norwegian harbours were selected for detailed investigations in the field and subjected to durability analysis. For each structure, the field investigation included a large number of chloride penetration measurements, electrochemical surface potential mapping and measurements of concrete cover. The equation used for modelling of the chloride penetration was based on Fick's Second Law of Diffusion in combination with a time dependent diffusion coefficient. The probability analysis of the durability performance was performed by use of a Monte Carlo Simulation.

For the most exposed beam elements, the durability analysis showed that the probability of failure in the form of onset of steel corrosion exceeded 10 % within a period of 10 to 15 years, while the field investigations revealed that the chloride front had already reached a depth varying from 40 to 50 mm after a service period of 7 to 8 years. For the most exposed anchor slab, a 10 % probability of failure was exceeded within a period of 10 years, while a depassivation of the embedded steel had already taken place after 8 years. If a probability-based durability design had been carried out as an integral part of the original design of the structures, such a poor durability would probably not have been acceptable.

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1. INTRODUCTION

Along the Norwegian coastline there are more than 9.000 concrete harbour structures, most of which are typically showing chloride-induced corrosion within a period of 5-10 years. As part of the general design, the durability is typically being specified in the form of some prescriptive requirements to maximum w/c-ratio and minimum cement content and concrete cover. Even for relatively new concrete structures, where the prescriptive durability requirements have been in accordance with the new European Concrete Code EN 206-1, premature corrosion of the embedded steel is being observed.

In order to obtain a more controlled durability and long-term performance of concrete structures in chloride containing environments, much research work has been carried out in recent years [1]. In particular, the development of new procedures for probability-based durability design has proved to be very valuable [2-4]. Although there is still a lack of relevant data, this methodology has already been successfully applied to a number of new concrete structures, where strict requirements to durability and long-term performance have been specified.

In order to provide more data and experience with probability-based durability design, two relatively new concrete structures in Norwegian harbours were selected for detailed investigations in the field and subjected to durability analysis.

2. DURABILITY ANALYSIS

2.1 Serviceability limit state

For reinforcement corrosion in a concrete structure, a degradation process as schematically shown in Figure 1 takes place.

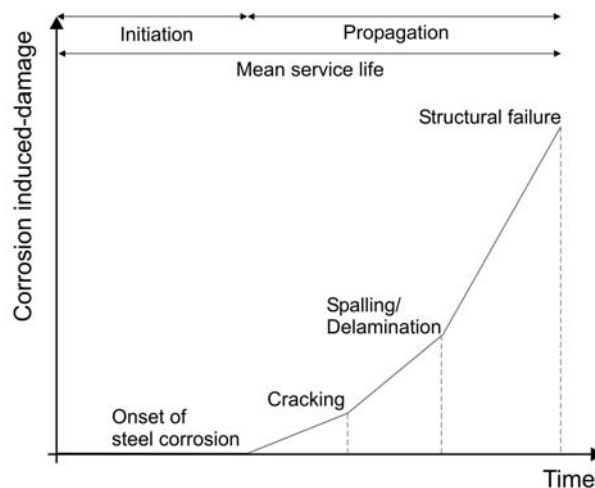


Figure 1. Degradation of a concrete structure due to reinforcement corrosion.

After depassivation or onset of steel corrosion, it may take several more years before any visual sign of corrosion such as cracking and spalling will occur (e.g. 3-5 years), and it may still take a very long time before the structural capacity of the structure becomes significantly reduced. However, since the time to depassivation represents both a critical and well defined stage of the deteriorating process, it appears appropriate to define this stage as the serviceability limit state in the durability analysis.

2.2 Chloride penetration

The rate of chloride penetration into concrete as a function of depth is normally modelled by the use of Fick's Second Law of Diffusion:

$$\frac{dC(x,t)}{dt} = D_C \frac{d^2C(x,t)}{dx^2} \quad (1)$$

where $C(x,t)$ is the chloride ion concentration at a distance x from the concrete surface after being exposed for a period of time t , and D_C is the chloride diffusion coefficient. By solving this differential equation for pre-defined boundary conditions, the following equation is obtained:

$$C(x,t) = C_S \left[1 - \operatorname{erf} \left(\frac{x}{2 \cdot \sqrt{Dt}} \right) \right] \quad (2)$$

where C_S is the chloride ion concentration on the concrete surface, and erf is the error function.

Since the diffusion coefficient is time dependant, a commonly used expression is [5]:

$$D(t) = D_0 \left(\frac{t}{t_0} \right)^n \quad (3)$$

where D_0 is the diffusion coefficient at the time t_0 , and the exponent n represents the ability of the concrete to increase the resistance against chloride penetration with time.

By substituting Eq. 3 into Eq. 2, an expression is obtained that permits the prediction of chloride levels based on the time dependent diffusion coefficient, given by:

$$c_x = c_{SC} \left[1 - \operatorname{erf} \left(0.5x / \sqrt{D_0 \cdot \left(\frac{t}{t_0} \right)^n t} \right) \right] \quad (4)$$

2.3 Probability analysis

In the present study, the probability analysis was calculated by use of a Monte Carlo Method (MCM) simulation. The estimation of the probability of failure was based on the evaluation of the limit state function for a large number of trials. The limit state function $g(r,s) < 0$, where s represents the load and r the resistance, is obtained by re-arranging Eq. 4. The load s can then be represented as the depth of chloride penetration:

$$x(t) = 2 \cdot \operatorname{erf}^{-1} \left(1 - \frac{c_{CR}}{c_S} \right) \cdot \sqrt{D_0 \cdot \left(\frac{t_0}{t} \right)^n \cdot t} \quad (5)$$

where c_{CR} is the critical chloride level at which the depassivation of the steel reinforcement occurs. The resistance r is defined by the depth of the concrete cover.

For each trial, the variables are randomly sampled from the probability distribution function that represents their scatter, and the limit state function $g(r-s) < 0$ is evaluated. The probability of failure is given by the ratio between the number of trials resulted in a negative performance of the limit state function and the total number of trials.

Since the accuracy of MCM mainly depends on the number of trials [6], and the method is easy to implement, a simulation based on MCM appear to be both simple and intuitive.

3. FIELD INVESTIGATIONS

Two relatively new concrete harbour structures from 1995 (S1) and 1996 (S2), respectively, were selected for detailed investigations in the field and subjected to a probability-based durability analysis. The structures had a waterfront of 131 and 80 m, respectively, and each structure consisted of an open concrete deck on top of steel tubes filled with concrete. Although durability requirements according to the Norwegian Concrete Code NS 3420 [7] with a maximum w/c-ratio of 0.45 and a minimum cement content of 300 kg/m³ and minimum concrete cover of 50 mm had been specified, approximately 400 kg/m³ of a high-performance Portland cement had been applied for both structures. Hence, the durability requirements according to the new European Concrete Code EN 206-1 [8] were also fulfilled. The specified concrete cover varied from 50 to 60 mm.

At the time of field investigation, the concrete structures S1 and S2 were 8 and 7 years old, respectively. The condition assessment was primarily based on a large number of chloride penetration measurements, surface potential mapping and concrete cover measurements. In order to characterize the level of concrete quality, accelerated chloride diffusivity and electrical resistivity were determined on drilled out concrete cores (Ø100 mm) by use of a migration test method [9]. Since the deck beams and the anchor slabs represented the most exposed and vulnerable parts of the structures, more detailed measurements were carried out on a few representative elements of each structure (Figure 2). For Structure S1, two beams (S1-B1 and S1-B2) and two sections of the anchor slab (S1-A1 and S1-A2) were subjected to more detailed investigations, while for Structure S2, one beam (S2-B3) and two sections of the anchor slab (S2-A3 and S2-A4) were investigated in more detail. For both structures, the dominant wind direction varied from NE to NW, while the top of the decks above mean water level was +3.00 m for S1 and +3.80 for S2, respectively.

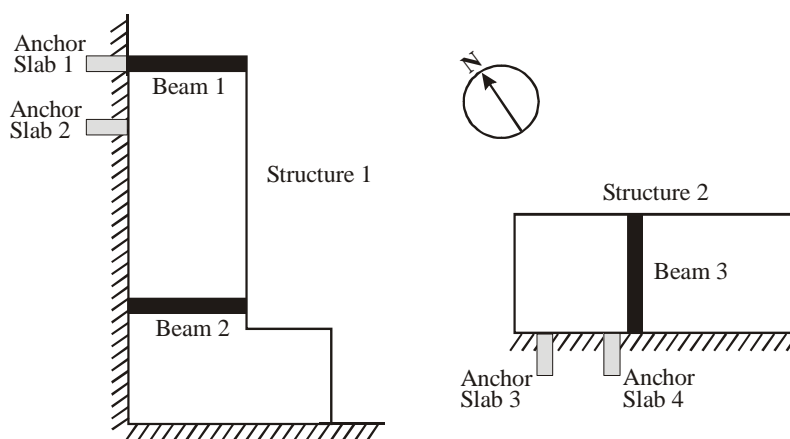


Figure 2. An overall view of the concrete harbour structures investigated.

4. RESULTS AND DISCUSSION

4.1 Condition assessment

For both concrete structures, the general condition appeared to be very good without any visual sign of reinforcement corrosion. For all the beams investigated, the critical level of chloride concentration (0.07% by weight of concrete) had not yet reached the level of

embedded steel (Figures 3 and 4), which had an average depth varying from 50 to 60 mm. Apart from S1-B2 where a chloride penetration of approximately 30 mm was observed, the critical level of chloride concentration had reached a depth varying from 40 to 50 mm.

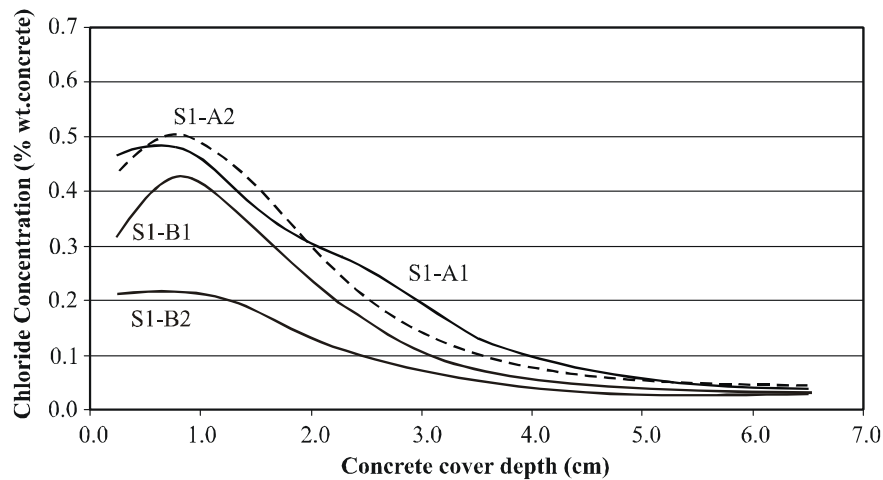


Figure 3. Average chloride penetration in Structure S1.

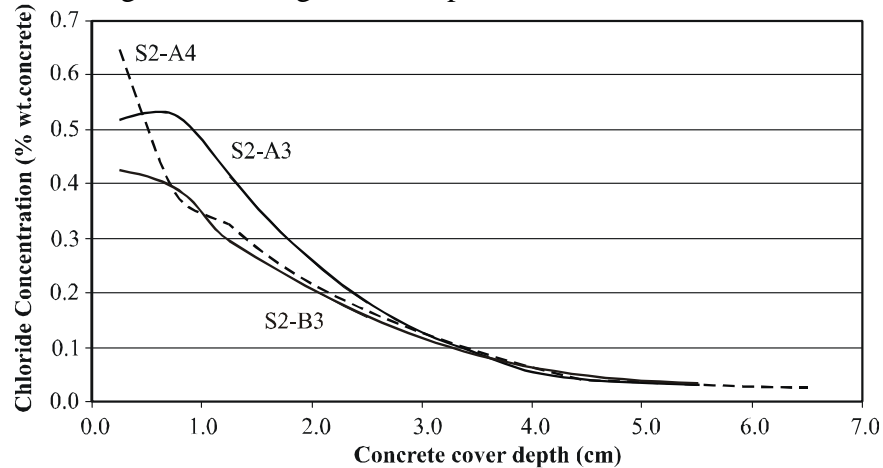


Figure 4 - Average chloride penetration in Structure S2.

The electrochemical potential mapping (EPM) did not reveal steel corrosion in any of the deck beams (Figure 5). For the anchor slabs, however, the EPM showed that a depassivation of the steel had already taken place (Figures 6 and 7), with the lowest negative potential values having gradients of -250 mV/m.

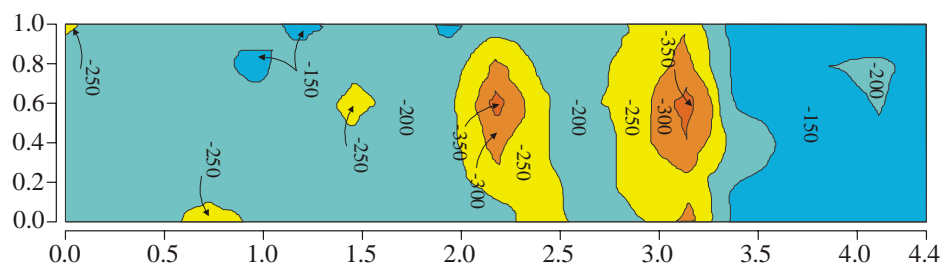


Figure 5. Electrochemical potential map of Beam S2-B3.

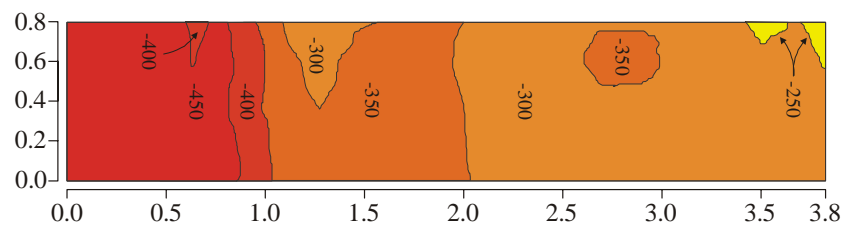


Figure 6. Electrochemical potential map of Anchor Slab S2-A3.

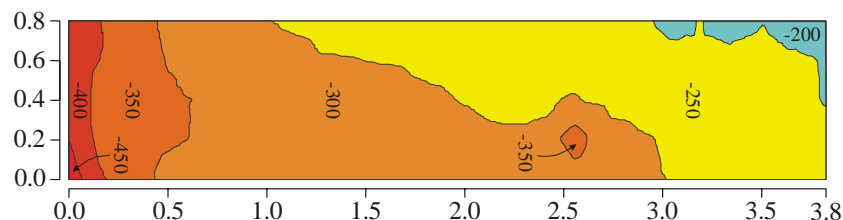


Figure 7. Electrochemical potential map of anchor slab S2-A4.

Based on previous experience [10], the observed test results on accelerated chloride diffusivity (D) and electrical resistivity (ρ) in Table 1 indicate that the concrete in both structures only had a moderate resistance against chloride penetration. These values were determined according to the NT Build 492 [9].

Table 1 – Chloride diffusivity and electrical resistivity of concrete in Structures S1 and S2 (normal distributed).

| Parameters | S1 (μ ; σ) | S2 (μ ; σ) |
|-------------------------------------|-------------------------|-------------------------|
| D (10^{-12} m ² /s) | (14; 0.1) | (10; 0.5) |
| ρ (Ω .m) | (42; 2.4) | (69; 2.8) |

In order to determine the surface chloride concentration (c_s) and the apparent diffusion coefficient (D_A), the chloride penetration curves were fitted with Eq. 2, the results of which are shown in Table 2 together with some other parameters for the durability analysis.

4.2 Durability analysis

In order to carry out the probability-based durability analysis, it was necessary to determine the statistical parameters which define the probability density curves of the model variables shown in Table 2. The concrete cover (x_c) for the structures was determined on the basis of a large number of concrete cover measurements. The apparent diffusion coefficients (D_A) were determined from the curve fitting of a large number of chloride penetration profiles. A critical chloride content (c_{CR}) of 0.07 % by weight of concrete with a CoV of 10 % was adopted from the literature, corresponding to a critical chloride concentration of 0.40 % by weight of cement for a concrete with 400 kg/m³ of portland cement. The surface chloride concentrations (c_s) were determined from the curve fitting of Fick's Second Law to the chloride profiles. The exponent n , which expresses the time-dependence of the diffusion coefficient, was based on a value suggested in the Duracrete Document BE95-1347/R15 [3] for Portland cement concrete in marine environment. The parameter for the model uncertainty takes into account the uncertainties, which are related to the simulation of the real degradation process [11].

Table 2. Statistical parameters for the durability analysis.

| Structure | S1 | | | | S2 | | |
|------------------------------------------------------|---------------|---------------|---------------|---------------|---------------|---------------|---------------|
| Element | B1 | B2 | A1 | A2 | B3 | A3 | A4 |
| x _C (mm) | N(53.3; 6.1)* | N(57.3; 4.2) | N(60.0; 5.0) | | N(60.8; 1.8) | N(60.0; 5.0) | |
| D _A (10 ⁻¹² m ² /s) | N(1.29;0.28) | | N(1.67; 0.28) | | N(1.81; 0.28) | N(1.23;0.26) | |
| c _{CR} (%/wt. conc.) | N(0.07;0.007) | | | | | | |
| c _S (%/wt conc.) | N(0.63; 0.10) | N(0.28; 0.09) | N(0.61; 0.07) | N(0.70; 0.05) | N(0.55; 0.13) | N(0.72; 0.09) | N(0.71; 0.03) |
| n (-) | N(0.37;0.07) | | | | | | |
| t ₀ (years) | D(8.0) | | | | D(7.0) | | |
| t (years) | D(50.0) | | | | | | |
| Model uncertainty | N(0.0;0.1) | | | | | | |

N - Normal distribution (average; standard deviation)

Based on the statistical parameters shown in Table 2, the probability of failure in the form of time to depassivation versus time of exposure was analysed for a period of up to 50 years as shown in Figures 8 and 9.

From Figure 8 it can be seen that the beams showed a difference in performance due to a difference in exposure conditions. Thus, Beam S1-B2 was typically more protected from exposure compared to that of the other beams. From Figure 9, the different performance of the anchor slabs also reflect that the anchor slab in Structure 2 was less exposed than that of Structure 1.

According to the Norwegian Standard "Requirements to reliability in design of structures"[12], the probability of failure for a serviceability limit state should not exceed 10 %, while according to Eurocode 1 [13], the probability of failure should not exceed 7 %. For the two most exposed beams investigated (S1-B1 and S2-B3), it can be seen from Figure 8 that the probability of failure in the form of depassivation exceeds 10 % within a period of 10 to 15 years, while the field investigations revealed that the chloride front had already reached a depth varying from 40 to 50 mm after a period of 7 to 8 years. For the most exposed anchor slab in Structure S1, where a 10 % probability of failure was reached after 10 years, a depassivation of the embedded steel had already taken place.

If a probability-based durability design had been carried out as an integral part of the original design of the structures, such a poor durability would probably not have been acceptable. A probability-based durability design would also have provided a basis for a performance-based quality control during concrete construction, and hence also, a basis for documentation of obtained construction quality, which would be of great importance for the future facility management of the structures [14].

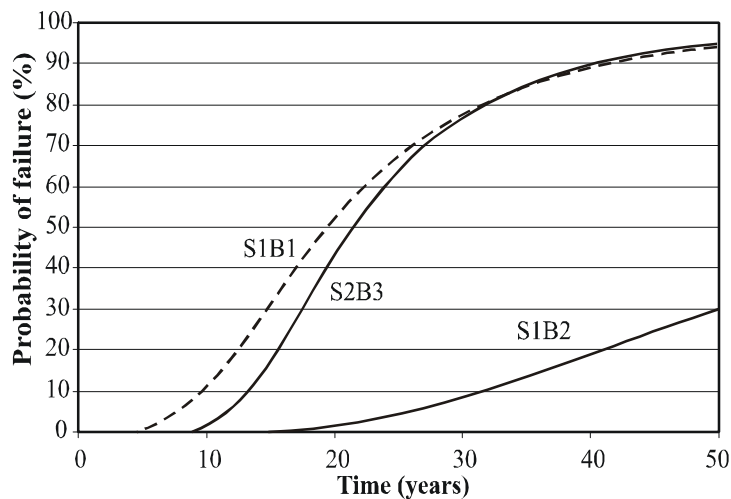


Figure 8. Probability of failure versus time of exposure for all the beams investigated.

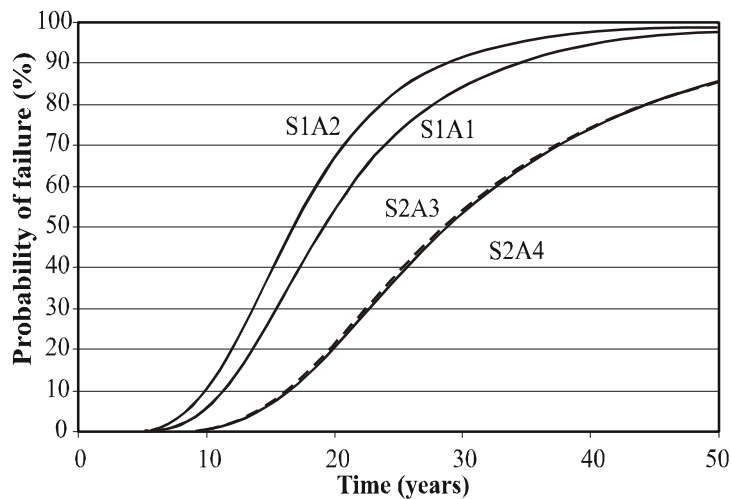


Figure 9. Probability of failure versus time of exposure for all the sections of the anchor slabs investigated.

5. CONCLUSIONS

Although both of the structures investigated are representative for relatively new concrete structures in Norwegian harbours, the present study was only based on these two structures and a limited amount of field measurements. It was also necessary to make a certain number of assumptions for the probability analysis. On the basis of the results obtained, however, the following conclusions appear to be warranted:

1. For the most exposed beam elements of the structures, the durability analysis showed that a 10 % probability of failure in the form of onset of steel corrosion would be exceeded already within a period of 10 to 15 years, while the field investigations revealed that the chloride front had already reached a depth varying from 40 to 50 mm after a service period of 7 to 8 years.

2. For the most exposed anchor slab, the durability analysis showed that a 10 % probability of failure would be exceeded already within a period of 10 years, while a depassivation of the embedded steel had already taken place after a period of 8 years.
3. For all new concrete structures in a chloride-containing environment where safety and high long-term performance are important, the present study shows that a probability-based durability design should be an important and integral part of the general design.

6. ACKNOWLEDGEMENTS

The first author would like to express his gratitude for the fellowship received from The Research Council of Norway during his staying at the Norwegian University of Science and Technology in Trondheim.

7. REFERENCES

- [1] Gjrv, O.E., "Durability and Service Life of Concrete Structures", Proceedings, The First fib Congress 2002, Session 8, V. 6, Japan Prestressed Concrete Engineering Association, Tokyo, 2002, pp. 1-16.
- [2] DuraCrete, "Modelling of degradation" The European Union – Brite EuRam III, Project No. BE95-1347, Probabilistic performance based durability design of concrete structures, Report No. R 4-5, 1998.
- [3] DuraCrete, "General guidelines for durability design and redesign", The European Union – Brite EuRam III, Project No. BE95-1347, "Probabilistic performance based durability design of concrete structures", Report No. T7-01-1, 1999.
- [4] McGee, R., "Modelling of durability performance of Tasmanian bridges", Proceedings, The 8th International Conference on the Application of Statistics and Probability, Sydney, Australia, 1999.
- [5] Takewaka, K. and Mastumoto, S., "Quality and Cover Thickness of Concrete Based on the Estimation of Chloride Penetration in Marine Environments", ACI SP 109-17, American Concrete Institute, 1988, pp. 381-400.
- [6] Di Sciuva, M. and Lomario, D., "A comparison between Monte Carlo and FORMs in calculating the reliability of a composite structure", *Composite Structures*, V. 59, 2003, pp. 155-162.
- [7] Norwegian Standard NS 3420, "Specifications for buildings and construction", Oslo, 1986.
- [8] EN 206-1, "Concrete – Part 1: Specifications, performance, production and conformity", CEN, 2000.
- [9] Nordtest Method NT Build 492, "Determination of chloride diffusion coefficient from non-steady state migration experiments".
- [10] Nilsson, L., Ngo, M.H. and Gjrv, O.E., "High-Performance Repair Materials for Concrete Structures in the Port of Gothenburg", Proceedings, Second International Conference on Concrete Under Severe Conditions – Environment and Loading, ed. by O.E. Gjrv, K. Sakai and N. Banthia, E & FN Spon, London and New York, V. 2, 1998, pp. 1193-1198.
- [11] Leira, B. J., Hynne, T., Lindgrd, J., "Marine Concrete Structures Subjected to Chloride Attack: Probabilistic Lifetime Assessment", Proceedings, The ETCE/OMAE2000 Joint

Conference: Energy for the New Millennium. American Society of Mechanical Engineers (ASME), New Orleans, USA, 2000.

- [12] Norwegian Standard NS-3490, "Design of structures – Requirements to reliability", Oslo, 1999.
- [13] EN 1991, "Eurocode 1: Basis of design and actions on structures", CEN, 2000.
- [14] GjØrv, O.E., "Durability Design and Construction Quality of Concrete Structures", Fourth International Conference on Concrete Under Severe Conditions – Environment and Loading, Seoul, Korea, June 2004, paper submitted.